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FUZZY LOGIC CONTROL OF BRIDGE STRUCTURES USING INTELLIGENT SEMI-ACTIVE SEISMIC ISOLATION SYSTEMS

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SUMMARY

Passive supplemental damping in a seismically isolated structure provides the necessary energy dissipation to limit the isolation system displacement. However, damper forces can become quite large as the passive damping level is increased, resulting in the requirement to transfer large forces at the damper connections to the structure which may be particularly difficult to accommodate in retrofit applications. One method to limit the level of damping force while simultaneously controlling the isolation system displacement is to utilize an intelligent hybrid isolation system containing semi-active dampers in which the damping coefficient can be modulated. The effectiveness of such a hybrid seismic isolation system for earthquake hazard mitigation is investigated in this paper. The system is examined through an analytical and computational study of the seismic response of a bridge structure containing a hybrid isolation system consisting of elastomeric bearings and semi-active dampers. Control algorithms for operation of the semi-active dampers are developed based on fuzzy logic control theory. Practical limits on the response of the isolation system are considered and utilized in the evaluation of the control algorithms.

The results of the study show that both passive and semi-active hybrid seismic isolation systems consisting of combined base isolation bearings and supplemental energy dissipation devices can be beneficial in reducing the seismic response of structures. These hybrid systems may prevent or significantly reduce structural damage during a seismic event. Furthermore, it is shown that intelligent semi-active seismic isolation systems are capable of controlling the peak deck displacement of bridges, and thus reducing the required length of expansion joints, while simultaneously limiting peak damper forces. Copyright © 1999 John Wiley & Sons, Ltd.

KEY WORDS: structural control; hybrid control; semi-active control; intelligent structures; fuzzy logic; seismic isolation

INTRODUCTION

Current design codes are under continuous evaluation in an effort to improve the safety of structures during a seismic event. However, the safety of structures during earthquakes usually

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does not apply to the structure itself but rather to life-safety. Most buildings are currently designed to yield in a strong seismic event, but not collapse, thus protecting the individuals within the structure. Yielding is permitted as a means of dissipating a portion of the energy transferred into the structure from the earthquake. However, plastic deformations due to yielding result in permanent damage and leave the structure susceptible to aftershocks. Thus, conventional structural design is often based on providing sufficient strength and stiffness to limit the inelastic deformations to an acceptable range.

An alternate seismic design philosophy is based on specifying the expected performance of the structural system during earthquakes. Thus, rather than simply ensuring life-safety during strong earthquakes, one may ensure minimal damage to the structural system as well. Seismic designs which follow this alternate design philosophy are known as performance-based seismic designs.[1] One way to meet the more stringent requirements of a performance-based seismic design is through the application of structural control. Structural control may be utilized to either reduce the amount of energy transferred into the structure from the ground motion or to absorb a portion of the earthquake energy after it has been transferred into the structure (see, for example, Reference 2). Structural control systems can be classified as either passive, active, semi-active, or hybrid.

Passive structural control systems require no external power or computer processes for operation. Since the control system properties cannot be modified after installation, the system is regarded as passive. Active control systems utilize actuators to apply control forces to the structure. The desired control forces are determined by incorporating the actuators within a feedback control system that utilizes the measured response of the structure or the measured ground motion as feedback. Active control systems often require a large power source for operation. Semi-active control systems may be regarded as passive control systems which have been modified to allow for the adjustment of mechanical properties. Thus, these systems may be regarded as having the adaptability characteristics of active control systems along with the reliability/stability characteristics of passive control systems. Semi-active control systems often require only enough power to operate a computer and a small electronic device for modifying mechanical properties. A comprehensive literature review of semi-active control systems has recently been completed by Symans.[3]

Hybrid control systems consist of combinations of the above-mentioned control systems. For example, passive hybrid seismic isolation systems typically consist of passive isolation bearings and passive energy dissipating devices. The isolation bearings lengthen the period of the structure to reflect a portion of the seismic input energy while the energy dissipating devices limit the isolation system displacement to an acceptable level. Such a system is currently being utilized within the Arrowhead Regional Medical Center in Colton, California.[4] Other types of hybrid control systems consist of active (e.g. see References 5–7) or semi-active (e.g. see References 8 and 9) control systems combined with passive control systems. For example, the research described herein involves an analytical and numerical study of the seismic response of a bridge structure employing a hybrid seismic isolation system consisting of passive seismic isolation bearings and semi-active dampers. The motivation for such a study arises from the fact that relatively large forces can develop in passive dampers and, correspondingly, large forces can be transmitted through the damper connections to the superstructure and substructure. Retrofitting weak structures to resist these forces can be difficult. It is envisioned that isolation systems that employ semi-active dampers will be capable of controlling the superstructure displacements while simultaneously limiting the design force for the damper and its connections.

FUZZY LOGIC CONTROL THEORY AND APPLICATIONS

A fuzzy logic feedback control system is employed within the intelligent semi-active seismic isolation system of the bridge structure analysed in this study. Thus, the fundamental aspects of fuzzy logic analysis and examples of previous applications of fuzzy logic analysis for control of semi-active seismic isolation systems are described in this section.

Fuzzy set theory was developed by Lotfi Zadeh[10] in 1965 to deal with the imprecision and uncertainty that is often present in real-world applications. The theory of fuzzy logic holds the premise that significance is more important than precision and, as a result, fuzzy set theory allows for human reasoning using verbose statements rather than mathematical equations to define the way an input is mapped to an output. This often results in complex input–output mapping relationships which, however, can easily be modified.

The primary difference between fuzzy logic and traditional mathematics is that fuzzy set theory allows objects to have a degree of membership within a set while traditional mathematics requires objects to have either 0 or 100 per cent membership within a set. As a result, fuzzy set theory involves terminology that is different from that of traditional mathematics. A brief description of fuzzy logic and fuzzy set terminology is provided below (the interested reader is referred to the general literature for more detailed discussions). The fuzzy logic analysis flowchart shown in Figure 1 should be referred to as a guide to the ensuing discussion. The flowchart is based on Mamdani's fuzzy inference method.[11] The first step in creating a fuzzy set is defining a *universe of discourse*. The universe of discourse is a finite or infinite range of values which define the range of non-fuzzy input and output values. The boundary of a fuzzy set is described by a *membership function* which is defined over a specified range on the universe of discourse. The purpose of membership functions is to define the degree of membership that a non-fuzzy number has within a fuzzy set. As mentioned above, a fuzzy set is a set whose objects do not require complete inclusion or exclusion (i.e. the objects can have a degree of membership). Membership functions can be of any shape as long as they represent a mathematical function, three of the most common being triangular, trapezoidal, and Gaussian. A fuzzy set which is 'normal' has a vertical range from 0 to 1 which signifies that a non-fuzzy number can have possible degrees of membership ranging from 0 to 100 per cent. Once defined, the membership functions receive a verbose name such as 'Large' or 'Fast'. A fuzzy logic system may have many membership functions which can, and usually do, overlap. This overlapping gives a non-fuzzy number partial membership in two or more different fuzzy sets. The development of a *fuzzy rulebase* follows the definition of the membership functions. Fuzzy rules are verbose sentences which define the way that the input membership functions are related to the output membership functions. These rules are in the form of IF … THEN … statements and are based on practical human reasoning which makes them relatively easy to understand. The rules can also be changed easily by only changing words, unlike that of a mathematical equation which can require changing many numbers or variables. The next stage is known as the *fuzzification* process where non-fuzzy numbers become fuzzy. This process is also known as fuzzy mapping, meaning that a non-fuzzy number is mapped onto the universe of discourse and is thus given a degree of membership for each of the membership functions. Using the fuzzy rulebase developed earlier, the *fuzzy implication* process occurs in which the input is mapped to the output membership functions. If there is more than one input corresponding to an output, a *fuzzy operation* is performed in which the degree of membership of the inputs are combined in preparation for fuzzy implication. The *fuzzy aggregation* process follows in which the output sets are combined onto a single universe of discourse. The final stage

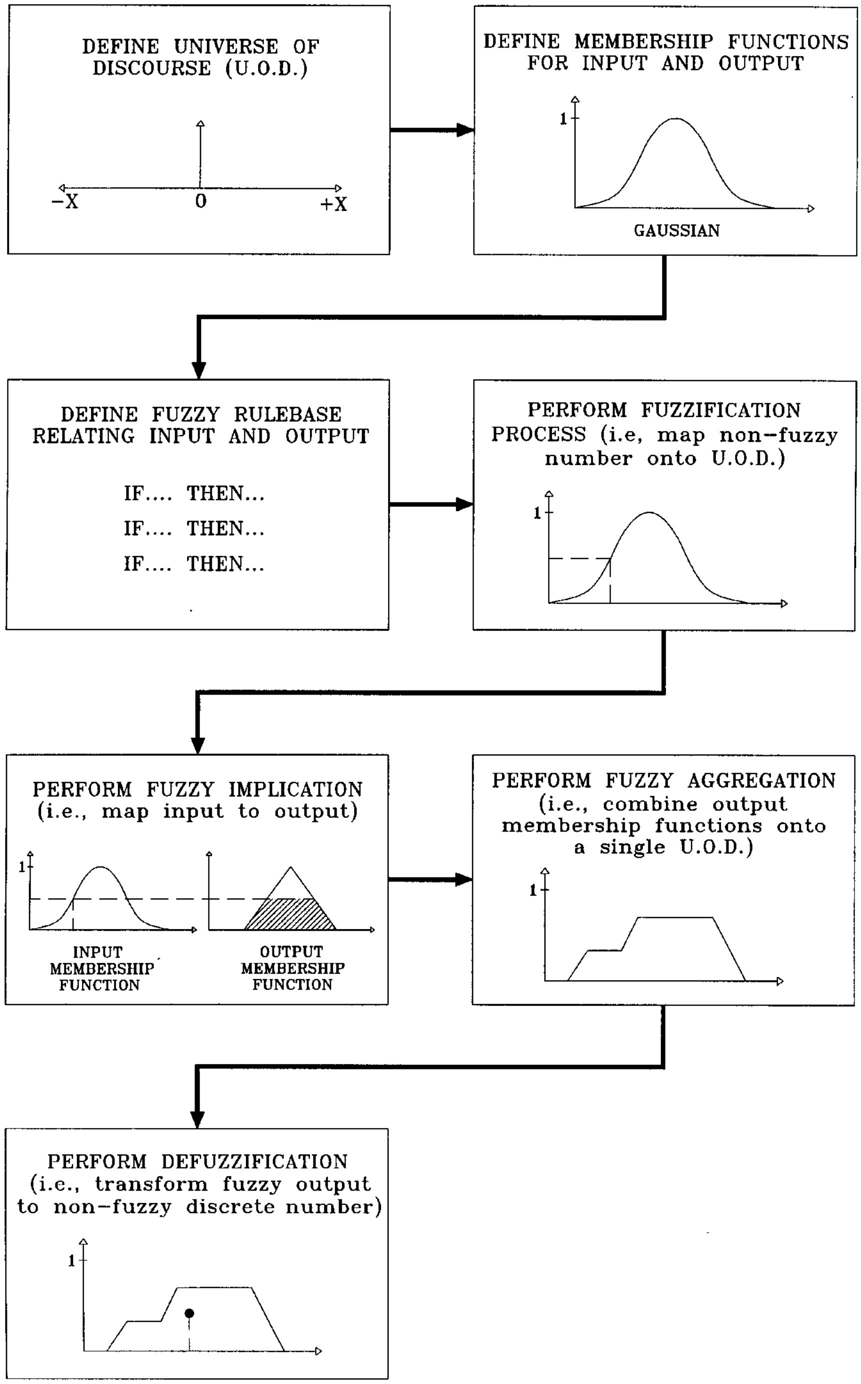

Figure 1. Flowchart for fuzzy logic analysis based on Mamdani's fuzzy inference method

of the fuzzy logic process is known as *defuzzification* wherein each fuzzy number is transformed to a non-fuzzy discrete output. At this point, the fuzzy logic process is complete. If the results are not as expected, different membership functions, implication, operation, aggregation, or defuzzification methods can be utilized to adjust the input–output mapping.

One of the most prominent uses for fuzzy logic in structural engineering applications is that of fuzzy control. As mentioned above, fuzzy logic is used to describe a complex mapping relationship between a set of inputs and outputs. Fuzzy logic control makes use of the mapping relationship to control the behaviour of a structural system. Fuzzy logic feedback control systems have been utilized in a number of structural control research studies due to the ability of such systems to work well with imprecise and uncertain data associated with environmental loads such as earthquake-induced ground motion and strong winds. The majority of fuzzy logic structural control research studies have been performed in the area of active control (e.g. see References 5–7 and 12–14), where the fuzzy controller is used to select the optimum force to be applied to the structure. There has been a limited amount of work in the area of semi-active control, where the fuzzy controller is used to vary the mechanical properties of a semi-active control device.

As an example of the application of fuzzy logic to the control of a semi-active device. Sun[8] employed fuzzy control theory to suppress bridge vibrations due to seismic loading using semi-active fluid dampers. Fuzzy control theory was used to determine the optimum damping level for the semi-active dampers. A three-span bridge model with semi-active dampers located between the bridge deck and bridge abutment was used for the analytical model. The fuzzy controller used damper force and damping ratio as input and the change in damping ratio as output. The dampers were pre-set to a high level of damping and the damping ratio was lowered if the maximum allowable damper force was to be exceeded. Sun[8] first explored the use of both low and high passive supplemental damping to control the bridge vibrations. In the case of passive low damping (i.e. damping ratio of 12 per cent), the deck displacement was reduced by about 27 per cent as compared to the uncontrolled bridge and the damper force was maintained below the the maximum allowable value. The uncontrolled bridge had a fundamental natural period of 2·3 sec and a corresponding damping ratio of 4·3 per cent. In the case of passive high damping (i.e. damping ratio of 100 per cent), the deck displacement was reduced by about 84 per cent, although the damper force exceeded the maximum allowable value. Next, Sun[8] explored the use of semi-active dampers providing a damping ratio which varied from 12 to 100 per cent. In this case, the bridge deck displacements were reduced by about 57 per cent and the damper force was maintained below the maximum allowable value. It is apparent from the data given above that the use of semi-active dampers offered a compromise in which the deck displacement was reduced significantly while the damper force was simultaneously limited to a prescribed allowable value. However, to more fully appreciate the difference between the performance enhancements that can be achieved with passive and semi-active damping control systems, it is necessary to also consider moderate levels of passive damping (as opposed to only low and high levels of passive damping as examined by Sun and Goto). This issue is explored in detail in the study described herein.

Another example of research that has been performed on semi-active fuzzy control system is that of Nagarajaiah,[9] who utilized fuzzy logic theory to control semi-active fluid dampers in a six-storey base-isolated structure. Based on input of relative velocity and displacement at the base of the structure, the semi-active damper force was output from the fuzzy controller. For the structure with passive dampers, the effective damping ratio was equal to 40 per cent (assuming that the structure above the isolation system vibrates as a rigid body). For the structure with semi-active dampers, the dampers were modulated such that the damping ratio varied up to 40

per cent. In general, the use of semi-active dampers produced the smallest base shear values, but larger displacements as compared to the use of linear and non-linear passive dampers. This result is expected since, in the semi-active control case, the damping ratio was always less than or equal to that of the passive control case. Thus, in the semi-active control case, the ability of the structural system to dissipate energy was generally less than for the passive control case and one might expect to observe larger displacements. Furthermore, the base shear forces in the passive control case are expected to be larger than in the semi-active control case since the forces transferred through the dampers will generally be larger due to the higher damping ratio in the passive control case.

Nagarajaiah's work raises the following important question. Could the damping ratio in the semi-active control case be allowed to increase beyond the value in the passive control case in such a way that the displacements are reduced below that achieved with passive control while simultaneously limiting the base shear force to a pre-selected value? This is essentially the question that is explored herein for the case of a seismically isolated bridge structure.

BRIDGE STRUCTURE AND SEISMIC EXCITATION

Description of bridge structure

The bridge structure utilized in this study represents a prototype-scale version of a 1:4-scale bridge model experimentally tested by Tsopelas[15] which consists of a single-span with a rigid deck and flexible piers (see Figure 2). The values of the parameters shown in Figure 2 are given in Table I. The bridge was designed so that the properties of the two piers could be different. This

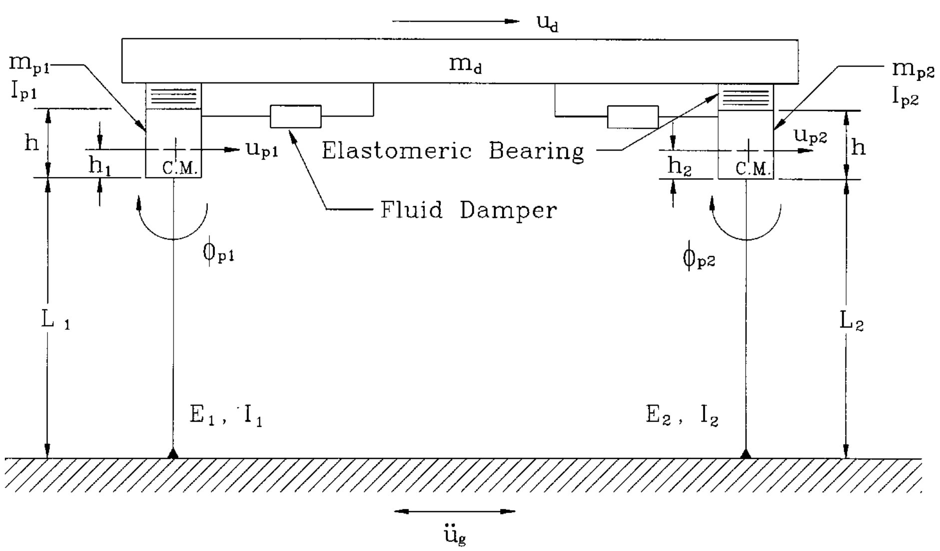

Figure 2. Bridge structure model in longitudinal direction (adapted from Reference 15)

Table I. Values of parameters for bridge model (prototype scale)

Parameter	Numerical value
Deck mass (m_d)	233·23 kN s^2/m
Pier mass (m_{p1}, m_{p2})	14·52 kN s^2/m
Pier column length (L_1, L_2)	6·4 m
Distance to pier cap centroid (h_1, h_2)	0·392 m
Height of pier cap (h)	1·652 m
Pier cap mass moment of inertia (I_{p1}, I_{p2})	9·78 kN s^2 m
Pier column modulus of elasticity (E_1, E_2)	200 000 MPa
Pier column moment of inertia (I_1, I_2)	0·007736 m^4

allows the bridge to be analysed as either an interior span of a multi-span bridge, a single span bridge on stiff abutments, or the first or last span of a multi-span bridge, where one pier would be flexible, and the other a stiff abutment. In this study, only the case of an interior span of a multi-span bridge (two flexible piers) was studied. The bridge isolation system consisted of two circular elastomeric bearings and a single fluid viscous damper located between the top of each pier cap and the bottom of the bridge deck. Studies on similar systems have been performed by Kawashima[16] and Yang.[17]

The piers of the bridge are fixed at the base and allowed to rotate freely at the top. In reality, there is some amount of rotational stiffness in the bearings which would resist this rotation. However, the study by Tsopelas[15] showed that the response of the structure could be reliably predicted while neglecting the rotational stiffness. Due to the flexibility of the Piers, the mathematical model of the bridge accounts for pier displacement and rotation in addition to deck displacement. The model also accounts for inherent damping in the piers and isolation bearings.

The displacement at which the pier columns yield was determined to assess the validity of the linear analysis performed in this study. If the pier column displacements exceed that corresponding to the yield point, the displacements from the linear analysis are not strictly correct. However, in the cases where yielding does occur, the displacements calculated from the linear elastic analysis can be used as a measure of performance. In particular, for structural systems having low natural frequencies, the approximate equivalence of the elastic and inelastic displacements has been regarded as generally valid (e.g. see Reference 18).

Equations of motion

The selected degrees of freedom are the relative deck displacement, u_d, the relative pier cap displacements, u_{pi}, and the pier cap rotations, ϕ_{pi} (see Figure 2). Note that the subscript variable i ($i = 1, 2$) indicates which pier is being referenced. In this study, the deck was assumed to be rigid, and the properties of the piers were assumed to be identical. Therefore, u_{p1} and ϕ_{p1} must equal u_{p2} and ϕ_{p2}, respectively. As shown in Figure 2, the pier caps are modelled as a lumped mass at the top of the columns. The lumped mass has height h, with the centre of mass located at a distance h_i from the bottom of the lumped mass. Pier cap rotations are assumed to be about this centre of mass. The following five equations of motion can be written to describe the deck translation and the pier cap translation and rotation:

$$m_d(\ddot{u}_d + \ddot{u}_g) + F_{I1} + F_{I2} = 0 \tag{1}$$

$$m_{p1}(\ddot{u}_{p1} + \ddot{u}_g - h_1\ddot{\phi}_{p1}) + F_{p1} - F_{I1} = 0 \tag{2}$$

$$m_{p2}(\ddot{u}_{p2} + \ddot{u}_g - h_2\ddot{\phi}_{p2}) + F_{p2} - F_{I2} = 0 \tag{3}$$

$$I_{p1}\ddot{\phi}_{p1} + M_{p1} + F_{p1}h_1 + F_{I1}(h - h_1) = 0 \tag{4}$$

$$I_{p2}\ddot{\phi}_{p2} + M_{p2} + F_{p2}h_1 + F_{I2}(h - h_2) = 0 \tag{5}$$

where u_g is the ground displacement, F_{Ii} is the lateral force in the isolation system, F_{pi} and M_{pi} are the shear force and bending moment, respectively, at the top of the pier columns, and the double overdots indicate second-order differentiation with respect to time. All other variables in equations (1)–(5) are defined in Table I. The lateral force in the isolation system consists of the forces in the elastomeric bearings and fluid dampers and can be written as

$$F_{Ii} = F_{bi} + F_{di} \tag{6}$$

where F_{bi} is the force in the elastomeric bearings and F_{di} is the force in the dampers. The bearings were modelled as simple Kelvin elements consisting of a linear elastic spring and a linear viscous dashpot in parallel. Thus, the bearing force may be written as

$$F_{bi} = K_{bi}u_{bi} + C_{bi}\dot{u}_{bi} \tag{7}$$

where K_{bi} is the bearing stiffness, u_{bi} is the bearing displacement, C_{bi} is the effective bearing damping coefficient, and the single overdot indicates first-order differentiation with respect to time. The fluid dampers were modelled as linear viscous dashpots with a constant damping coefficient (passive control case) and a variable damping coefficient (semi-active control case) as given by the following expressions:

$$F_{di} = C\dot{u}_{di} \quad \text{(passive damping)} \tag{8}$$

$$F_{di} = C(t)\dot{u}_{di} \quad \text{(semi-active damping)} \tag{9}$$

where C is a constant damping coefficient, $\dot{u}_{di}$ is the damper velocity which is equal to the bearing velocity, $\dot{u}_{bi}$, and $C(t)$ is a variable damping coefficient which is bounded by a minimum and maximum value. Dampers which exhibit the behaviour described by equations (8) and (9) have been experimentally tested by Constantinou[19] and Symans,[20,21] respectively. Since the top of the pier columns are assumed to rotate freely, the lateral force and bending moment at the connection of the pier cap to the top of the column can be represented as follows:

$$\begin{Bmatrix} F_{pi} \\ M_{pi} \end{Bmatrix} = E_i I_i \begin{bmatrix} \dfrac{12}{L_i^3} & \dfrac{6}{L_i^2} \\ \dfrac{6}{L_i^2} & \dfrac{4}{L_i} \end{bmatrix} \begin{Bmatrix} u_{pi} \\ \phi_{pi} \end{Bmatrix} + \begin{bmatrix} C_{pi}^1 & 0 \\ 0 & C_{pi}^2 \end{bmatrix} \begin{Bmatrix} \dot{u}_{pi} \\ \dot{\phi}_{pi} \end{Bmatrix} \tag{10}$$

where C_{pi}^1 is the inherent damping in the columns due to lateral motion and C_{pi}^2 is the inherent damping in the columns due to pier rotation. In this study, C_{pi}^1 is taken to be the damping coefficient corresponding to a damping ratio of 5 per cent in the fundamental mode of vibration of the piers and C_{pi}^2 is regarded as negligible and thus is set equal to zero. These values of C_{pi}^1 and C_{pi}^2 are consistent with the experimental results obtained by Tsopelas.[15] Using the relationships given by equations (6)–(10), the equations of motion given by equations (1)–(5) can be written in terms of the five selected degrees of freedom (for details, see Reference 22) in preparation for the ensuing state-space analysis.

A non-isolated bridge structure was also examined to evaluate the benefits of including an isolation system. The non-isolated bridge structure was idealized to have columns which are fixed at the base and are free to rotate at the pier caps (i.e. the bridge deck is simply supported) resulting in a natural period of 0·54 s.

Seismic isolation bearing design

The elastomeric seismic isolation bearings were designed using the single-mode spectral analysis method of the AASHTO[23] 'Guide Specifications for Seismic Isolation Design'. The design resulted in a natural period of 2·2 s in the fundamental mode and a design displacement of 26·47 cm. The bearings consisted of alternating rubber and steel layers having a total height of 35·25 cm and a diameter of 55·88 cm. The bearing design satisfies the AASHTO[23] Guide Specifications for shear strain capacity and vertical load stability. Details of the bearing design are provided in Reference 22.

Seismic excitation

Recorded ground motions from six historical earthquakes were selected as input for this study (see Table II). The earthquake records were selected to represent a wide range of ground motion characteristics in terms of, for example, frequency content and peak motion. The ground motion characteristics shown in Table II were compiled from various sources and verified using digitized ground acceleration records. In particular, the predominant frequency range was determined using the half-power bandwidth method as described by Kramer.[24] In this paper, selected results for input being the 1940 Imperial Valley earthquake (El Centro record—component S00E), the 1995 Hyogo-Ken Nanbu earthquake (Kobe City record—component NS) and the 1971 San Fernando earthquake (Pacoima Dam record—component S16E) will be presented. Detailed results from analyses with all six earthquakes are provided in the thesis by Kelly.[22]

FUZZY LOGIC FEEDBACK CONTROL SYSTEM

The fuzzy logic feedback control system was developed within a simulation model using SIMULINK® (vs. 1·3),[25] a dynamic analysis and simulation software program which uses MATLAB® (vs. 4·2)[26] for its mathematical calculations. SIMULINK® is ideal for structural control applications since it permits the construction of simulation models in block diagram form. The Fuzzy Logic Toolbox (vs. 1·0)[27] is an additional software program for use with SIMULINK® and MATLAB® which permits a fuzzy logic controller to be placed within a SIMULINK® model. A brief description of the method for creating the simulation models is described below.

The equations of motion as given by equations (1)–(5) were written in state-space form for analysis within SIMULINK®. The state-space representation of the system is given by

$$\begin{aligned}\{\dot{x}\} &= [A]\{x\} + [B]\{f\}\\ \{y\} &= [C]\{x\}\end{aligned} \tag{11}$$

Table II. Summary of earthquake site and ground motion characteristics

Earthquake	Station	Epicentral distance (km)	Site geology	Magnitude	Predo-minant freq. range (Hz)	Peak accel. (g)	Peak velocity (cm/s)	Peak displ. (cm)
Imperial Valley May 18, 1940	El Centro Comp. S00E	12	Alluvium	6·7	0·5–2·8	0·34	33·45	10·87
Kern County July 21, 1952	Taft Lincoln School Tunnel Comp. N21E	41	Alluvium (40ft) Over Sandstone	7·2	0·5–3·3	0·16	15·72	6·71
San Fernando Feb. 9, 1971	Pacoima Dam Comp. S16E	3	Rock	6·4	0·4–4·6	1·17	113·23	37·67
Northridge Jan. 17, 1994	Sylmar Parking Lot Comp. 90°	16	Alluvium	6·8	0·5–2·5	0·60	76·94	15·22
Hyogo-Ken Nanbu Jan. 17, 1995	Kobe City Comp. NS	20	Alluvium	6·9	0·6–2·7	0·83	90·84	22·02
Michoacan Sept. 19, 1985	SCCT (Mexico City) Comp. N90W	373	Soft Clay	8·1	0·3–0·6	0·17	60·50	21·20

where $\{x\}$ is the state vector which fully defines the state of the system, $[A]$ is the system matrix, $[B]$ is the input location matrix, $\{f\}$ is the input vector containing the effect of the ground acceleration, $\ddot{u}_g$, $\{y\}$ is the output vector, and $[C]$ is the output location matrix. Recall that control of the bridge structure was achieved through a seismic isolation system consisting of elastomeric bearings and fluid dampers. The fluid dampers provide damping which may be either passive (fixed) or semi-active (variable). In the semi-active case, the variable damping coefficient was modified based on response feedback from the structural system. The effect of the fluid damper control force was accounted for by including the variable damper force as an additional component of the input vector, $\{f\}$, of equation (11). The variable dampers were controlled using both fuzzy logic control theory and a simple non-linear control algorithm.

The design of the fuzzy controller began with selection of the response quantities to be used as input to the fuzzy controller and the distribution and type of membership functions to be used for the selected inputs. The fuzzy control system input was either a displacement, velocity, or acceleration response quantity. A uniform distribution of nine trapezoidal membership functions was used for the fuzzy input as shown in Figure 3(a). The definitions of the fuzzy input membership function abbreviations are as follows: NVL = Negative Very Large, NL = Negative Large, NM = Negative Medium, NS = Negative Small, ZO = Zero, PS = Positive Small, PM = Positive Medium, PL = Positive Large and PVL = Positive Very Large. A reasonable range of input values (i.e. $-X$ to $+X$ in Figure 3(a)) must be selected for the input membership functions since, if the range is too large, the outermost membership functions will rarely be utilized and thus limit the variability of the control system. Conversely, if the range is too small, the outermost membership functions will essentially be utilized at all times, again limiting the variability of the control system. The range of values must also be established for the output membership functions which consisted of a uniform distribution of five triangular membership functions as shown in Figure 3(b). The definitions of the output membership function

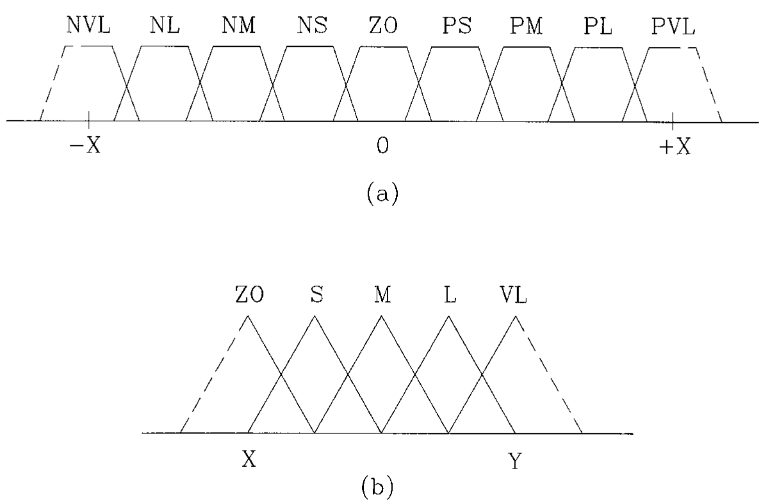

Figure 3. Input and output membership functions for fuzzy control analysis

abbreviations are as follows: ZO = Zero, S = Small, M = Medium, L = Large, VL = Very Large. The damping coefficient of the semi-active dampers is the fuzzy control system output for the structural control system and thus the parameters X and Y in Figure 3(b) represent the minimum and maximum damping coefficient, respectively.

CONTROL CASES AND FUZZY RULEBASES

The response of the bridge structure was evaluated for the case of a non-isolated bridge deck and for the case of an isolated bridge deck with a hybrid isolation system consisting of elastomeric bearings and either passive or semi-active fluid dampers. In the cases where passive dampers were utilized, the damping ratio in the fundamental mode of the structure was set to various values ranging from 2 to 100 per cent. In the cases where semi-active dampers were utilized, two different configurations were employed. In the first configuration, the damping ratio in the fundamental mode was allowed to vary from 2 to 40 per cent by modulating the variable damping coefficient of equation (9). In an effort to enhance the effect of damping further while simultaneously considering the force demand on the semi-active dampers (and the associated connections to the structure), a second configurations was investigated in which a constraint was placed on the maximum output force of the semi-active dampers while simultaneously permitting the damping ratio in the fundamental mode to vary from 2 to 80 per cent. Since the semi-active dampers in this study did not represent a specific type of damper with a prescribed force capacity, a reasonable damper force limit was estimated. The damper force limit was selected to be equal to the maximum force achieved in the numerical simulations for the isolated bridge with passive dampers providing a damping ratio of 40 per cent in the fundamental mode. The limiting force of 304·5 kN was obtained in the 1971 San Fernando earthquake.

A total of eight fuzzy rulebases were developed for controlling the bridge structure. Rulebases 4, 5 and 7 are described herein since they typically produced the most significant response reductions. Details of the other five rulebases may be found in Reference 22. Note that fuzzy rulebases 1–6 were those in which the semi-active dampers were modulated such that the damping ratio varied from 2 to 40 per cent while fuzzy rulebases 7 and 8 were those in which the semi-active dampers were modulated such that the damping ratio varied from 2 to 80 per cent while simultaneously constraining the damper force to a prescribed limit.

Rulebase 4 was developed to simulate an on–off (bang–bang) control algorithm. For such a control algorithm, the damping coefficient is binary in that it can be either high or low. A heuristic description of rulebase 4 is as follows. When the bridge deck is moving toward its centre position with respect to the pier cap (i.e. sign of isolation system velocity opposite to sign of displacement), lower the damping coefficient and thus damping force, to allow it to return with minimal resistance. In contrast, when the bridge deck is moving away from its centre position (i.e. sign of isolation system velocity is equal to sign of displacement), increase the damping coefficient, and thus damping force, to constrain its motion. The fuzzy rulebase matrix (i.e., the input–output membership function relation matrix) and the control surface (i.e. the three-dimensional non-fuzzy input–output relationship) are shown in Figure 4. Note that with two inputs (isolation system displacement and velocity) and one output (damping coefficient), the fuzzy rulebase matrix is composed of 81 rules. The bang–bang nature of the fuzzy controller is clearly evident in the control surface. Furthermore, note that since the damping coefficient can only take on two values, the rulebase matrix is composed entirely of two output membership functions (zero (ZO) and very

FUZZY RULEBASE 4

Displacement

Velocity

C	NVL	NL	NM	NS	ZO	PS	PM	PL	PVL
PVL	ZO	ZO	ZO	ZO	VL	VL	VL	VL	VL
PL	ZO	ZO	ZO	ZO	VL	VL	VL	VL	VL
PM	ZO	ZO	ZO	ZO	VL	VL	VL	VL	VL
PS	ZO	ZO	ZO	ZO	VL	VL	VL	VL	VL
ZO	VL	VL	VL	VL	VL	VL	VL	VL	VL
NS	VL	VL	VL	VL	VL	ZO	ZO	ZO	ZO
NM	VL	VL	VL	VL	VL	ZO	ZO	ZO	ZO
NL	VL	VL	VL	VL	VL	ZO	ZO	ZO	ZO
NVL	VL	VL	VL	VL	VL	ZO	ZO	ZO	ZO

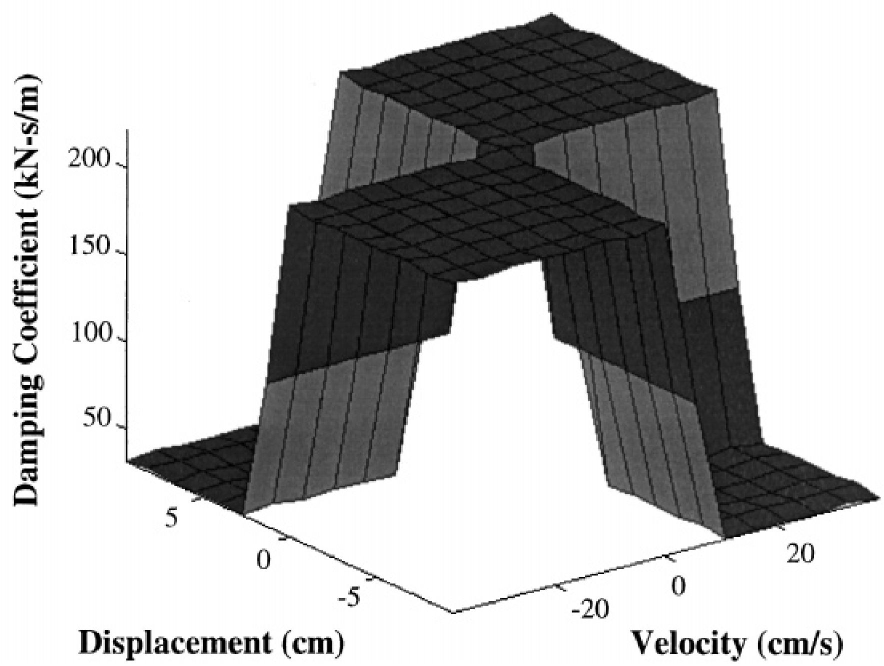

Figure 4. Fuzzy rulebase matrix and control surface for rulebase 4

large (VL)). An example of the interpretation of the fuzzy rulebase is as follows: If the velocity is negative small (NS) and the displacement is negative large (NL), then the damping coefficient is very large (VL). It should be noted that, due to the simple input–output relationship of rulebase 4, the rulebase could also have been implemented without resorting to fuzzy logic methods.

Rulebase 5 is similar to rulebase 4 except that the damping coefficient is gradually reduced as the structure is returning to the centre position as compared to the rapid reduction characterized by rulebase 4. The gradual reduction in damping coefficient, and thus damping force, is intended to reduce the acceleration response of the bridge deck as compared to that obtained with rulebase 4. The fuzzy rulebase matrix and control surface are shown in Figure 5. Note that it is apparent from an examination of the control surfaces in Figures 4 and 5 that rulebase 5 may be regarded as a smoothed version of rulebase 4. Further, note that, in contrast to rulebase 4, since the damping coefficient can take on multiple values, the rulebase matrix is composed of all five output membership functions.

The concept behind rulebase 7 is to apply high damping forces at low isolation system velocities and low damping forces at high isolation system velocities. In this way, the energy dissipation over a range of velocities is appreciable while the damping force is simultaneously constrained to a prescribed limit. As shown in Figure 6, the damping coefficient is smoothly reduced from its largest value to its smallest value as the magnitude of the velocity increases from zero to 100 cm/s. Note that rulebase 7 utilizes only the isolation system velocity as input, thus reducing the number of fuzzy rules from 81 to 9 and the control surface to a two-dimensional curve.

In addition to the eight fuzzy control algorithms, a simple non-linear control algorithm was investigated. This algorithm is similar to rulebases 7 and 8 in that the damping coefficient of the semi-active dampers was modulated so as to permit the damping ratio in the fundamental mode to vary from 2 to 80 per cent while simultaneously constraining the force output of the dampers to a prescribed limit. The algorithm was implemented by maintaining the damping coefficient of the dampers at a value corresponding to a damping ratio of 80 per cent for low damper velocities and reducing the damping coefficient as the magnitude of the velocity increased in such a way as to maintain the highest level of damping possible while simultaneously limiting the damper force output to a prescribed value. As shown in Figure 7, the resulting damping coefficient is a non-linear function of the damper velocity. A comparison of Figures 6 and 7 reveals that the non-linear control algorithm may be regarded as a modified version of fuzzy rulebase 7.

SEISMIC SIMULATION RESULTS

The seismic simulation results are presented in this section followed by a discussion of results in the following section. The primary objective of the simulations was to assess the possible advantages of utilizing semi-active dampers in a seismic isolation system as compared to the use of passive dampers. Numerous other studies have already demonstrated that passive damping within seismic isolation systems can be used to control isolation system displacements. This study confirmed those results by comparing the results of the seismic isolation case with passive dampers to the non-isolated case. A secondary objective of this study was to determine what trends occurred in the seismic response of a structure as the passive damping level was modified over a wide range. The effectiveness of both the passive and semi-active control systems

FUZZY RULEBASE 5

Displacement

Velocity

C	NVL	NL	NM	NS	ZO	PS	PM	PL	PVL
PVL	L	VL	VL	VL	VL	VL	VL	VL	VL
PL	M	L	VL	VL	VL	VL	VL	VL	VL
PM	S	M	L	VL	VL	VL	VL	VL	VL
PS	ZO	S	M	L	VL	VL	VL	VL	VL
ZO	VL	VL	VL	VL	VL	VL	VL	VL	VL
NS	VL	VL	VL	VL	VL	L	M	S	ZO
NM	VL	VL	VL	VL	VL	VL	L	M	S
NL	VL	VL	VL	VL	VL	VL	VL	L	M
NVL	VL	VL	VL	VL	VL	VL	VL	VL	L

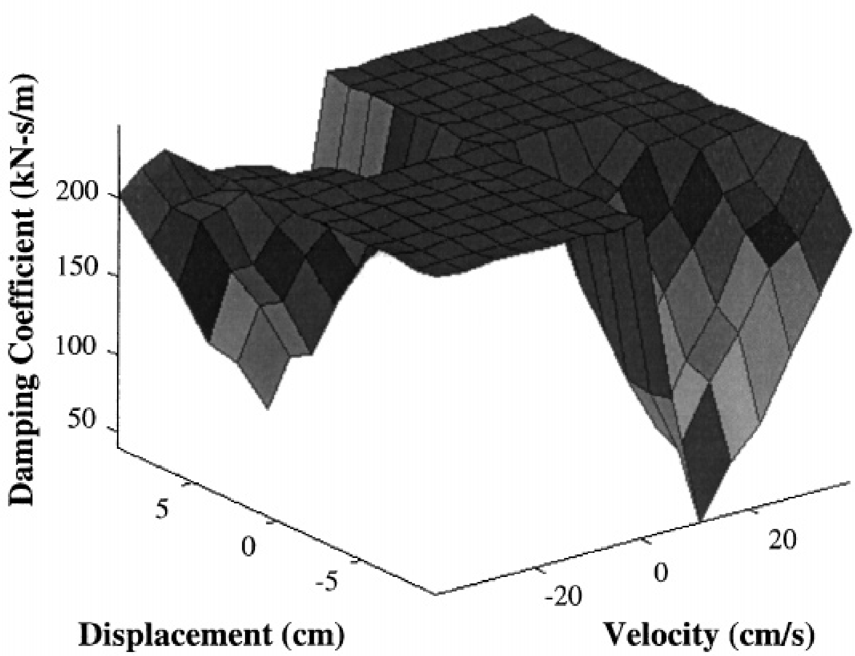

Figure 5. Fuzzy rulebase matrix and control surface for rulebase 5

was evaluated based on peak values of the following six response quantities: relative deck displacement (i.e. displacement between deck and ground), bearing displacement, pier drift ratio, base shear coefficient (i.e. base shear-to-weight ratio), absolute deck acceleration, and damper force.

FUZZY RULEBASE 7

Velocity

	NVL	NL	NM	NS	ZO	PS	PM	PL	PVL
C	ZO	S	M	L	VL	L	M	S	ZO

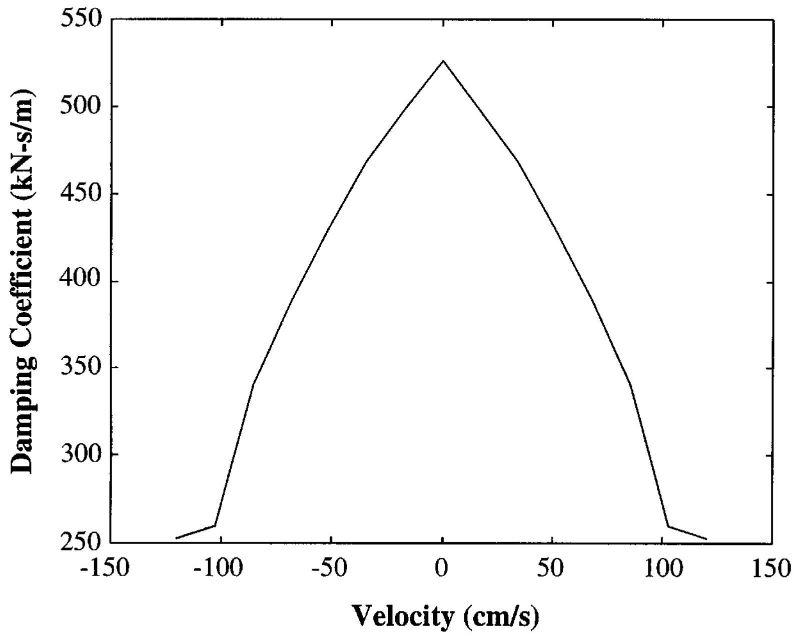

Figure 6. Fuzzy rulebase matrix and control surface for rulebase 7

Passive isolation system

Selected results from the analysis of the structure with a passive damping isolation system are shown in Figure 8. The peak bearing displacement is reduced most significantly as the damping ratio in the fundamental mode is increased from 0 to 40 per cent with minor additional reductions beyond 40 per cent (see Figure 8(a)), the peak pier drift generally decreased as the damping ratio increased to 40 per cent and, in some cases, increased moderately as the damping ratio was increased beyond 40 per cent (see Figure 8(b)), and the peak damper force monotonically increased as the damping ratio increased from 0 to 100 per cent (see Figure 8(c)). Note from Figure 8 that the bearing stability and pier yielding limits were exceeded for some of the earthquakes for low damping ratios. In addition, at high damping ratios the damper force limit was exceeded for some of the earthquakes. Based on a visual inspection of the results shown in

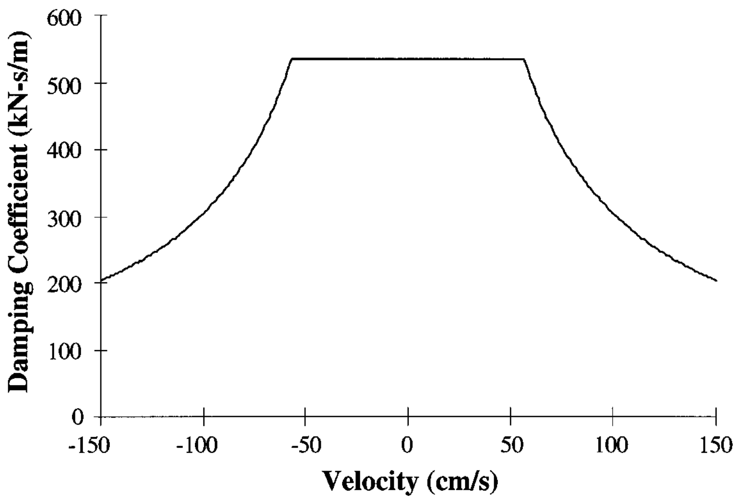

Figure 7. Damping coefficient variation for nonlinear control algorithm

Figure 8, it was concluded that, for the six earthquakes considered in this study, a fundamental mode damping ratio of about 40 per cent may be regarded as *optimal* in the sense that the bearings remain stable, the piers remain elastic and the damper force is not excessive. Note that the word *optimal* has been italicized to emphasize that the definition given above is not based on a rigorous analysis.

Semi-active isolation system

The objective of utilizing semi-active dampers was to improve the structural response as compared to that obtained using passive dampers providing an *optimum* level of damping wherein, as explained above, the *optimum* level was regarded as that corresponding to damping ratio of 40 per cent in the fundamental mode. Therefore, peak values of each response quantity of interest are evaluated for the non-isolated case, the isolated case with passive damping of 40 per cent, and the isolated case with semi-active damping. In addition, results for the isolated case with passive damping of 80 per cent are provided as a measure of the performance of the seismic isolation system with high passive damping. Typical comparisons of results for input being the 1940 Imperial Valley earthquake, the 1995 Hyogo-Ken Nanbu earthquake and the 1971 San Fernando earthquake are shown in Tables III–V, respectively. The data shown in the top and middle portion of the tables provides a comparison between the non-isolated case, the passive damping case with a damping ratio of 40 per cent and 80 per cent, and the semi-active damping cases for fuzzy rulebases 1–6. Recall that fuzzy rulebases 1–6 were those in which the semi-active dampers were modulated such that the damping ratio varied from 2 to 40 per cent. The data shown in the bottom portion of the tables provides results for the semi-active damping cases in which fuzzy rulebases 7 and 8 and the non-linear control algorithm were utilized. Recall that fuzzy rulebases 7 and 8 and the non-linear control algorithm were those in which the semi-active dampers were modulated such that the damping ratio varied from 2 to 80 per cent while

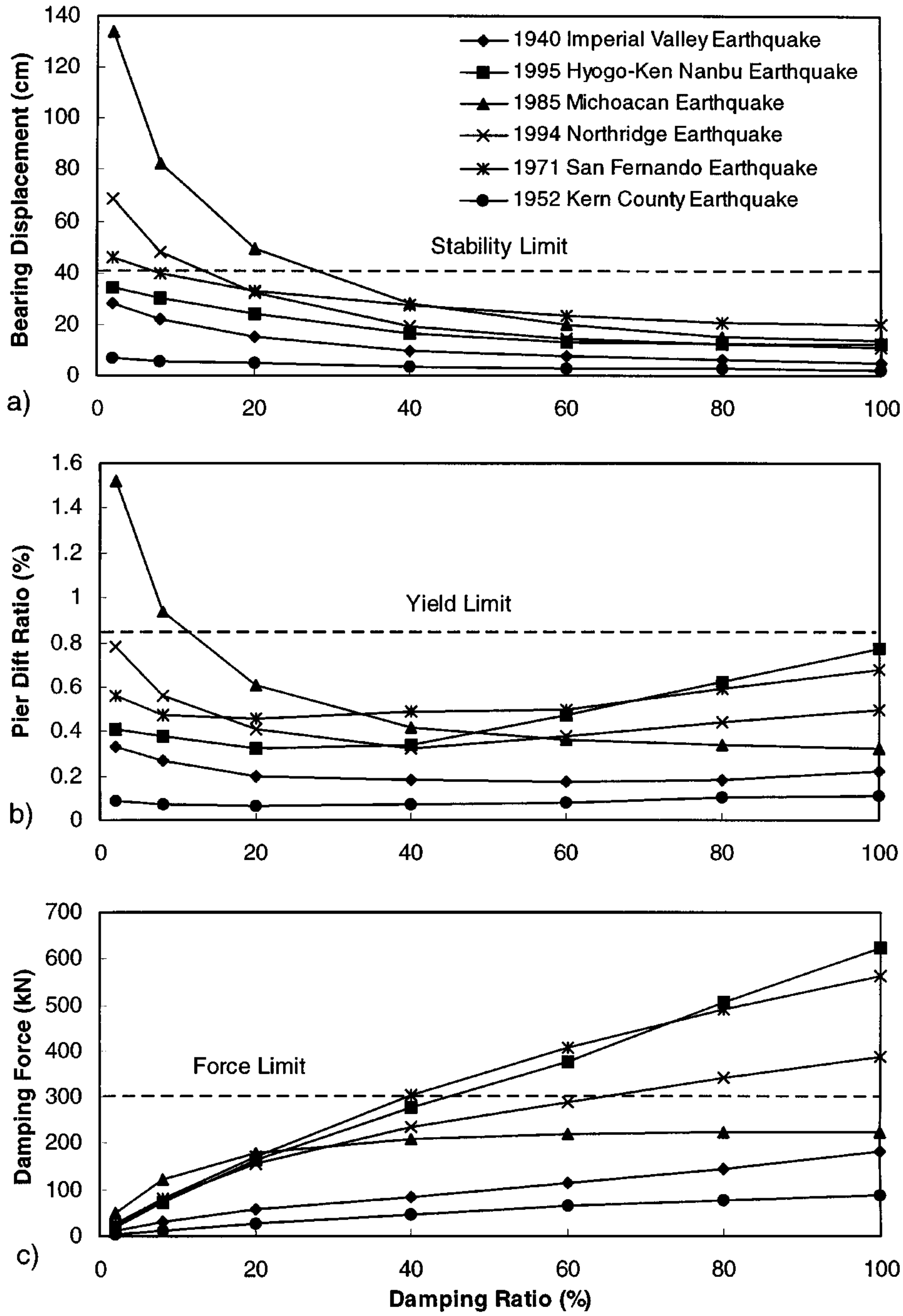

Figure 8. Peak response of bridge with passive seismic isolation system

Table III. Peak response of bridge structure subjected to the 1940 Imperial Valley Earthquake (El Centro Record—Component S00E)

Control method	Normalized peak response: Deck displacement	Bearing displacement	Pier drift ratio	Base shear coefficient	Deck acceleration	Damper force
Non-isolated	0·64	—	5·89*	6·27	4·95	—
Passive 40%	10·62 cm	9·86 cm	0·18%	0·15	0·282 g	0·27
Passive 80%	0·61	0·59	1·00	1·07	0·94	0·47
Fuzzy 1	1·80	1·80	1·39	1·47	1·06	0·14
Fuzzy 2	1·65	1·65	1·28	1·33	1·05	0·14
Fuzzy 3	1·34	1·34	1·17	1·27	1·07	0·28
Fuzzy 4	1·33	1·33	1·11	1·20	1·11	0·29
Fuzzy 5	1·13	1·13	1·00	1·07	1·02	0·26
Fuzzy 6	1·64	1·66	1·17	1·20	1·05	0·14
Fuzzy 7	0·64	0·63	1·06	1·07	**0·94**	0·44
Fuzzy 8	0·75	0·74	**0·94**	**1·00**	**0·94**	0·38
Nonlinear	**0·61**	**0·59**	1·00	1·07	**0·94**	0·47

Notes: (1) All response quantities except damper force are normalized with respect to the *optimal* passive 40% case
(2) Damper force is normalized with respect to the prescribed damper force limit of 304·5 kN
(3) Largest response reductions achieved by semi-active control system are indicated in boldface type
(4) * indicates that the prescribed limit has been exceeded
(5) Bearing Displ. Limit = 40·89 cm, Pier Drift Ratio Limit = 0·84 per cent, Damper Force Limit = 304·5 kN

Table IV. Peak response of bridge structure subjected to the 1995 Hyogo-Ken Nanbu Earthquake (Kobe City Record—Component NS)

Control method	Normalized peak response: Deck displacement	Bearing displacement	Pier drift ratio	Base shear coefficient	Deck acceleration	Damper force
Non-isolated	1·12	—	9·15*	9·13	11·00	—
Passive 40%	17·72 cm	16·43 cm	0·34%	0·30	0·319 g	0·91
Passive 80%	0·80	0·76	1·82	1·83	1·32	1·66
Fuzzy 1	1·67	1·67	1·09	1·07	1·10	0·33
Fuzzy 2	1·65	1·66	1·06	1·07	1·10	0·33
Fuzzy 3	1·39	1·40	1·26	1·27	1·11	0·71
Fuzzy 4	1·48	1·49	1·29	1·27	1·11	0·71
Fuzzy 5	1·09	1·09	**0·97**	**0·97**	**0·99**	0·86
Fuzzy 6	1·80	1·82	1·09	1·10	1·30	0·29
Fuzzy 7	**0·81**	**0·78**	1·26	1·27	1·13	1·00
Fuzzy 8	0·87	0·87	1·21	1·23	1·04	0·99
Nonlinear	0·82	**0·78**	1·38	1·37	1·15	1·00

Notes: (1) All response quantities except damper force are normalized with respect to the *optimal* passive 40% case
(2) Damper force is normalized with respect to the prescribed damper force limit of 304·5 kN
(3) Largest response reductions achieved by semi-active control system are indicated in boldface type
(4) * indicates that the prescribed limit has been exceeded
(5) Bearing Displ. Limit = 40·89 cm, Pier Drift Ratio Limit = 0·84 per cent, Damper Force Limit = 304·5 kN

Table V. Peak response of bridge structure subjected to the 1971 San Fernando Earthquake (Pacoima Dam Record—Component S16E)

Control method	Normalized peak response: Deck displacement	Bearing displacement	Pier drift ratio	Base shear coefficient	Deck acceleration	Damper force
Non-isolated	0·46	—	4·27*	4·28	3·67	—
Passive 40%	29·37 cm	27·33 cm	0·49%	0·43	0·739 g	1·0
Passive 80%	0·76	0·74	1·20	1·21	0·96	1·61
Fuzzy 1	1·35	1·36	**0·92**	0·93	1·00	0·37
Fuzzy 2	1·34	1·34	**0·92**	**0·91**	1·00	0·36
Fuzzy 3	1·21	1·21	1·16	1·16	1·03	0·99
Fuzzy 4	1·25	1·25	1·14	1·16	1·04	0·99
Fuzzy 5	1·03	1·03	0·98	1·00	1·01	0·97
Fuzzy 6	1·47	1·48	**0·92**	0·93	1·00	0·37
Fuzzy 7	0·83	0·82	1·04	1·05	0·97	1·00
Fuzzy 8	0·90	0·89	1·02	1·02	0·98	0·99
Nonlinear	**0·80**	**0·78**	1·08	1·09	**0·96**	1·00

Notes: (1) All response quantities except damper force are normalized with respect to the *optimal* passive 40% case
(2) Damper force is normalized with respect to the prescribed damper force limit of 304·5 kN
(3) Largest response reductions achieved by semi-active control system are indicated in boldface type
(4) * indicates that the prescribed limit has been exceeded
(5) Bearing Displ. Limit = 40·89 cm, Pier Drift Ratio Limit = 0·84 per cent, Damper Force Limit = 304·5 kN

simultaneously constraining the damper force to a prescribed limit. Note that, except for the damper force, all peak response values shown in Tables III–V are normalized with respect to the results for the isolated bridge with the *optimum* level of passive damping. The peak damper force shown in Tables III–V is normalized with respect to the prescribed force limit of 304·5 kN. Also, note that response quantity limits corresponding to pier column yielding, elastomeric bearing stability, and damper force capacity are indicated in Tables III–V. Furthermore, the maximum response reduction achieved by the semi-active isolation systems as compared to the *optimal* passive isolation system are indicated in boldface type.

DISCUSSION OF RESULTS

As mentioned previously, the bridge response was monitored to check for the occurrence of inelastic behaviour. Such behaviour occurred primarily for the case of the non-isolated bridge and thus it may be concluded that the use of both the passive and semi-active hybrid isolation systems was beneficial in preventing yielding in the pier columns (see Tables III–V). However, for the 1985 Michoacan earthquake inelastic behaviour occurred in the isolated bridge case with low passive damping levels (see Figure 8(b)). This result is to be expected since the 1985 Michoacan earthquake (Mexico City record) primarily consists of harmonic-type motion at a period of about 2 s which nearly coincides with the fundamental period of 2·2 s for the isolated bridge structure. Thus, for this earthquake, the bridge resonates and for low damping, the resonant response is

expected to be quite large.[28] For the 1985 Michoacan earthquake, the peak value of all response quantities, with the exception of damper force, continued to decrease with increasing passive damping levels (see Figure 8). This is due to the fact that, for steady-state harmonic motion, increases in damping tend to monotonically reduce the response with the most significant reductions occurring near resonance.[28]

The simulations for the bridge structure with a passive seismic isolation system revealed that the *optimum* level of damping corresponded to a damping ratio of 40 per cent in the fundamental mode of vibration. For this level of damping, a compromise is reached in which the motion of the structure is small enough to prevent inelastic pier column behaviour and bearing instability and the forces developed in the dampers is not excessive (see Figure 8). For levels of damping beyond 40 per cent, the damper forces can become quite large, leading to possible difficulties in the application of passive damping systems to seismic retrofit projects. Furthermore, one may note in Figure 8(b) that, in some cases, the pier drift ratio begins to rise beyond some level of damping. For example, when the isolated bridge is subjected to the 1995 Hyogo-Ken Nanbu earthquake, the pier drift ratio increases by 82 per cent as the damping ratio increases from 40 to 80 per cent (see Figure 8(b) and Table IV). In this particular case, the bridge piers remained elastic for all passive damping levels. However, it is apparent from Figure 8(b) that both very low and very high levels of damping may generally induce inelastic pier behaviour. This effect of extreme levels of passive damping may be explained as follows. At very low damping levels, the isolation system displacement tends to be large (see Figure 8(a)) and thus induces relatively large forces in the isolation bearings which, in turn, are transferred to the pier caps. These large forces may induce inelastic pier behaviour. At very high damping levels, the isolation system displacement tends to be small (see Figure 8(a)) due to high damping forces that effectively act to lock the bridge deck to the bridge piers. As such, an isolated structure with very high passive damping levels conceptually behaves more like a non-isolated structure in which the inertia forces associated with the deck motion induce large forces in the piers resulting in inelastic pier column behaviour. In summary, moderate levels of passive damping within seismic isolation systems appears to be more desirable than either very low or very high levels of passive damping.

In general, the semi-active damping control cases in which the fundamental mode damping ratio was limited to 40 per cent (i.e. fuzzy rulebases 1–6) produced significant increases or minor reductions in peak response as compared to the *optimal* passive damping case (although the bridge piers remained elastic) (see Tables III–V). This result is a reflection of the general difficulty one may find with improving the seismic response of structures that utilize well-designed passive control systems. As shown in Tables III–V, the bang–bang fuzzy control rulebase (rulebase 4) generally produced the largest accelerations of the bridge deck for the fuzzy rulebases in which the damping ratio was limited to 40 per cent (i.e. fuzzy rulebases 1–6). This is to be expected since the bang–bang controller results in abrupt changes in the damping coefficient (and thus damping force) of the semi-active dampers (see Figure 4). The abrupt change in damper force results in large accelerations. As expected, the smoothed fuzzy control rulebase (rulebase 5) results in smaller peak accelerations as compared to rulebase 4 (see Tables III–V). Furthermore, rulebase 5 tended to produce the largest reductions (or smallest increases) in peak response for the fuzzy rulebases in which the damping ratio was limited to 40 per cent. Thus, it is apparent that the smoothed bang–bang controller of Figure 5 offers an advantage over the standard bang–bang controller of Figure 4.

In general, the semi-active damping control cases in which the damping ratio was allowed to increase to 80 per cent while simultaneously constraining the damping force (i.e. fuzzy rulebase

7 and 8 and the non-linear control algorithm) produced significant reductions in peak deck and bearing displacements along with significant increases or minor reductions in peak deck acceleration, pier drift ratio, and base shear coefficient as compared to the *optimal* passive damping case (although the bridge piers remained elastic) (see Tables III–V). As expected, the semi-active damper forces were larger than in the *optimal* passive damping case since the damping ratio was permitted to increase above 40 per cent (except for the 1971 San Fernando earthquake from which the prescribed damper force limit was established). Interestingly, the non-linear control algorithm in which the damping coefficient was set to the highest possible value without exceeding the prescribed damper force limit did not produce the smallest response for the Hyogo-Ken Nanbu earthquake (see Table IV). In this case, fuzzy rulebase 7 resulted in smaller peak response values for all response quantities (except bearing displacement), albeit the difference in peak response values was not very significant.

Focusing on the peak deck displacement, it may be seen from Tables III–V that none of the semi-active isolation systems with damping ratios limited to 40 per cent produced lower peak deck displacements as compared to the *optimal* passive damping case. In fact, some of the semi-active fuzzy controllers produced peak deck displacements that were nearly double that corresponding to the *optimal* passive damping case. However, for the semi-active isolation systems in which the damper force was limited and the damping ratio was allowed to increase to 80 per cent, the peak deck displacement was always less than that corresponding to the *optimal* passive damping case. This result is significant in that it indicates that intelligent semi-active hybrid isolation systems are capable of controlling peak deck displacements, and thus reducing the required length of expansion joints, while simultaneously limiting peak damper forces and thus ensuring the integrity of both the damper itself and its connections to the bridge deck and pier.

SUMMARY AND CONCLUSIONS

The effectiveness of a hybrid isolation system for seismic protection of a bridge structure has been investigated in this study. The isolation system was designed using standard code-based procedures, thus ensuring that system implementation was feasible. Further, physical constraints on the isolation system (i.e. shear strain limits, stability limits, and damper force limits) were considered. In general, the study showed that hybrid seismic isolation systems consisting of combined base isolation bearings and supplemental energy dissipation devices can be beneficial in reducing the seismic response of structures. These hybrid systems may prevent or significantly reduce structural damage during a seismic event.

Passive supplemental damping in a seismically isolated structure provides the necessary energy dissipation to limit isolation system displacements which can be significantly increased when a structure is isolated. Further, peak values of structural response generally show a significant reduction as fundamental mode damping ratios are increased to about 40 per cent with no significant effect beyond a damping ratio of about 40 per cent. In contrast, damper forces can become quite large as the damping ratio is increased to large values, resulting in the requirement to transfer large forces at the damper connections to the structure which may be particularly difficult to accommodate for retrofitted structures. One method for limiting the level of damping force, while simultaneously ensuring sufficient damping to control the structural response, is to utilize an intelligent hybrid seismic isolation system containing semi-active dampers in which the

damping coefficient can be modulated over a wide range of values. In addition to limiting the level of damping forces, such systems are capable of controlling the peak deck displacement of bridges, and thus reducing the required length of expansion joints.

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